

The application of endurance time method for optimum seismic design of steel moment-resisting frames using the uniform deformation theory

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Abstract:

The optimum seismic design of structures is one of the biggest issues for engineers to build resistant and economic structures. In this research, the application of the endurance time method in optimum performance design of steel moment-resisting frames using the uniform deformation method is evaluated. First, three steel moment-resisting frames with 3, 7 and 12 stories are considered. After that, the structures are optimized by endurance time method analysis and the uniform deformation theory, under a series of acceleration functions. Also, results are compared with the results of time history analysis based on earthquakes. The results revealed that endurance time method and time history analysis of earthquakes at low and moderate seismic hazard levels are well matched, while this adjustment does not exist for high seismic hazard level. In addition, the optimum structure at one hazard level does not lead to optimum structure in other hazard levels. To have the best performance at different hazard levels, the frames should be optimized at the moderate seismic hazard level. In order to optimize the structure at all seismic hazard levels, the GAP dampers can be used. These dampers should be effective after a specified drift at the lower seismic hazard level. In addition, the best values for convergence power of the uniform deformation method are between 0.05 to 0.15 for this purpose. By using such dampers, it is possible to have uniform drift distribution at different seismic hazard levels.

1. Introduction

The proper seismic design of structures is one of the biggest challenges in building structures with enough strength, stiffness and ductility. The recent progress in earthquake engineering and dynamic behavior of structures has revealed the weaknesses of conventional methods which are based on force control concepts. Therefore, a lot of effort has been dedicated to find the most reliable and reasonable methods [1]. Studies have shown that the seismic design based on displacement control is more reasonable than the design based on force control, due to the fact that structure damage is mostly caused by deformations [2]. The lateral force distribution has an important role in structural design and the deformation distribution.

The distribution is completely dependent on the earthquake and structural properties; therefore, using the force distribution based on the codes will not necessarily lead to a proper design with a suitable seismic performance [3-5]. Cannon and Klink [6] proposed a method for calculating bending and shear stiffness distribution of structures by solving the equation of motion for elastic systems. Pezeshk et al. [7] proposed a method for optimum design of steel frames using a genetic algorithm according to AISC provisions. Shukla and Datta [8] studied the performance of steel structures equipped with viscoelastic dampers and presented a method for their optimum design. The proposed distribution of dampers showed that the designed structures based on this method exhibited less drift and response compared with other distributions of dampers. Karami [9] suggested a method for strength distribution pattern in structures using the uniform deformation theory which can decrease damage and increase the efficiency of material usage. According to the uniform deformation theory, a

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structure in which all of its stories reach a predefined value of deformation at a specific seismic hazard level is more economical than a structure in which some of its stories reach this value [10].

In recent years, many studies on proper distribution of structural elements at different stories have been carried out to optimize seismic performance, and different methods have been proposed for this purpose [11], [12]. Moghaddam and Hajrasouliha [13] showed that using the uniform deformation theory, a structure can be designed for a specific earthquake record which behaves better than the equivalent weight designed structure based on the proposed methods in seismic codes. Karami *et al.* [14] and Moghaddam and Karami [15] showed that the behavior of structures against dynamic loads are highly affected by the initial loading pattern. Improper loading pattern selection leads to inappropriate design and the structural design based on a specific earthquake record does not ensure proper structural behavior for other earthquakes. Therefore, in recent years, most of the codes consider their design criteria based on structural performance.

In performance-based design, seismic hazard levels depending on the likelihood of occurrence, are divided into weak, moderate and severe categories, and for each one of these levels, the specific performance of the structure is expected. For example, structural design is done in a way that the structures in low earthquakes have immediate occupancy level, life safety performance level in medium earthquakes and collapse prevention level in severe earthquakes [16]. In an ideally performance-based design, structures should be designed in such a way that at all levels of seismic hazard, the performance of the structure is in full compliance with the performance objectives, and the capacity of the structure is fully utilized. Lee and Goel [17], proposed an effective way to design based on performance using the target drift. They showed that the shear force caused by an earthquake does not always correspond to the shear force of the proposed load pattern of the code and designing by conventional code methods does not lead to proper use of lateral load resistance elements. Hajrasouliha *et al.* [18] suggested an efficient method for optimum design of concrete frames at different seismic hazard levels by combining the performance-based design and the uniform deformation theory. They showed that the designed structures suffer less damage during earthquake compared to the same-weight structure designed by IBC-2009 [19]. Karami and Ghasemof [20] compared performance-based design with the uniform deformation theory and heuristic algorithms and showed that the uniform deformation theory can be used as an efficient tool in performance-based seismic design due to its high computational speed in finding the optimal structure. Karami and Sharghi [21] proposed a practical method based on the uniform deformation theory

for optimization of eccentrically braced steel frames. Mogaddam and Gelekolai [22] proposed a method for the optimal cross-section distribution of structural elements in steel moment-resisting frames using the uniform deformation theory and an adaptive method in order to attain the lowest damage due to earthquakes. The results showed that a more uniform deformation under earthquakes and less weight in comparison to original structures can be obtained. Ganjavi and Hajrasouliha [23] presented a method for optimizing the concentrated braced steel frames subjected to near-fault ground motions using the uniform deformation theory and their results were validated by conducting nonlinear dynamic analysis. Nabid *et al.* [24] investigated the convergence rate and computational efficiency of optimization method for using friction dampers in reinforced concrete frames using the concept of uniform distribution of deformation. The reliability of the results was also compared to heuristic optimization methods. Karami *et al.* [25] developed a model based on the uniform distribution of deformation for optimum strengthening of steel moment frames using buckling restrained braces. Asadi and Hajrasouliha [26] proposed a practical method based on the concept of the uniform damage theory, in which the total life-cycle cost is considered as the main objective function for optimum seismic design of concrete frames. Their results showed that all predefined performance targets are satisfied and the maximum inter-story drift ratio and total life cycle cost of the frames are reduced. Gao and Li [27] presented the optimum seismic design method for reinforced concrete frame structures in which the longitudinal reinforcement of columns is modified to obtain uniform distribution of damage along the height of the building using incremental dynamic analysis.

In the conventional performance design of structures, a set of records are used in order to define seismic hazard levels for time history analysis [28], [29]. Investigating the structural behavior under these sets of records and controlling the intended performance criteria is time-consuming. Therefore, a new method called endurance time method was suggested by Estekanchi *et al* [30]. Endurance time method is a type of time history analysis based on dynamic pushover in which the structure is affected by an increasing dynamic excitation, whose intensity gradually increases over time. The structural response over time, which is proportional to different seismic intensities is investigated, and considering the corresponding response to different levels of excitation intensity, the pros and cons and performance of the structure are evaluated [31]–[35]. The main advantage of the endurance time method is to examine the behavior of structures at different seismic levels and it can be a good alternative for seismic analysis of structures in linear and nonlinear range [36]. In addition, its results are in good agreement with other conventional seismic analysis

methods, which is shown in separate studies [30], [37]. Efforts have been made to optimally generate endurance time excitations using wavelet theory [38], particle swarm optimization method [39], as well as considering probabilistic distribution parameters [40]. A large amount of effort has been put in for application of endurance time method for performance-based design or other structural analyzes, for example, to investigate the interaction of moment frame and shear wall [41]. Mirzaee *et al.* [42] investigated the performance-based design of steel frames using endurance time method. They studied different frames with different stories and showed that their engineering demand parameters (story drifts and plastic hinge rotations) in different levels of performances could be obtained with good accuracy and low calculation time using endurance time method. Hariri-Ardebili [43] investigated the seismic behavior of steel moment-resisting frames using endurance time method, time history analysis and incremental dynamic analysis. They showed that endurance time analysis can predict the general trend of IDA curves accurately. Rahimi and Estekanchi [44] investigated the collapse potential of buildings using endurance time method. The results showed that endurance time method is in good agreement with incremental dynamic analysis and seismic fragility curves can be easily produced with it. Estekanchi and Basim [45] proposed a method based on the proper distribution of the viscous dampers at the height of the building in order to improve the performance of the structures under different seismic levels simultaneously. They used the genetic algorithm in their research to optimize the structure. Foyouzat and Estekanchi [46] investigated the seismic performance of steel structures equipped with energy dissipating devices using endurance time method. Amouzegar *et al.* [47] optimized the damper properties in structures using incremental dynamic analysis and endurance time method. Mirfarhadi and Estekanchi [48] proposed a method for optimal seismic design of structures considering maximum value as the design objective. The results are verified and compared to the results of incremental dynamic analysis. It was shown that the value-based design approach significantly increases the total economic value. Recently, a review of endurance time method, its concepts and applications has been carried out by Estekanchi *et al.* [49].

In this research, by combining the uniform deformation theory and endurance time method, which are fast and accurate methods, a method is proposed for designing a structure that provides uniform deformation to the structure at all desired seismic hazard levels and utilizes the material's capacity more optimally. This method is executed for steel moment-resisting frames and the performance of the designed structures is evaluated using time history analysis for the earthquake record sets at the respective hazard levels.

2. Methodology

Three steel moment-resisting frames with 3, 7 and 12 stories are used in order to investigate our proposed method. These frames are modeled in OpenSees software [50]. The beams and columns are modeled in two ways. In the first method, beams and columns are modeled using a nonlinear beam-column element. In this method, the fiber model is used for modeling the geometry of the section and the number of integration points along each element is equal to 5. In this model, the cross-section in any form is divided into small rectangular shapes. The fiber section is utilized in order to create section geometry and by separating the sections into smaller parts, it is possible to study the plastic behavior of members more accurately. In the second method, the beams and columns are modeled using an elastic beam-column element and these elements are connected to each other with the concentrated plastic hinges. In order to model these concentrated hinges, a zero-length spring of type Steel 01 is added between two points with the same coordinates. Ibarra and Krawinkler [51] suggested that the initial elastic stiffness of the springs and the beam-column elements are determined by Eqs. (1-3)

$$K_s = nK_{bc} \quad (1)$$

$$K_{bc} = \frac{n+1}{n} K_{mem} \quad (2)$$

$$K_s = (n+1)K_{mem} \quad (3)$$

where K_s is the elastic stiffness of the spring, n is the correction factor in which the value is 10 according to Ibarra and Krawinkler suggestion, K_{bc} is the beam or column stiffness and K_{mem} is the member stiffness. Taking Eq. (4) as the stiffness of the member, the stiffness of the spring is considered as Eq. (5)

$$K_{mem} = \left(\frac{6EI}{L}\right)_{mem} \quad (4)$$

$$K_s = \left(\frac{6EI}{L}\right)_{mem} (n+1) \quad (5)$$

According to Eq. (6), the value of yield strength of the spring is equal to the plastic moment of the member.

$$M_p = ZF_y \quad (6)$$

where M_p is the plastic moment, Z is plastic section moduli and F_y is the yield stress of the material. In addition, the strain hardening of the spring is calculated by Eq. (7) according to Ibarra and Krawinkler suggestion

$$\alpha_s = \frac{\alpha_{mem}}{n+1-n\alpha_{mem}} \quad (7)$$

In Eq. (7), α_s shows the strain hardening value for the spring and α_{mem} defines the strain hardening of the member. It is worthwhile noting that the beam and the column section properties should be modified according to Eq. (8)

$$I_{bc} = \frac{n+1}{n} I_{mem} \quad (8)$$

Since in this model the beams and columns' connection to the plastic joints are in series, the overall element rotation in the plastic range is considered to be calculated by Eq. (9)

$$\theta_{mem} = \theta_s + \theta_{bc} = \frac{M}{K_s} + \frac{M}{K_{bc}} \quad (9)$$

Since the elastic stiffness of the spring in the Ibarra and Krawinkler models is ten times the stiffness of the beam and the column, the amount of spring rotation in the elastic domain is 10% of the column and beam elastic rotation. Therefore, in this study, the amount of rotation of the member in the elastic range is assumed to be equal to Eq. (9). This will in fact increase the elastic rotation of the members to 10% more in the elastic range. Fig. 1 shows the connections of the beams and columns and the plastic hinges in a sample frame with one span and two floors. In Fig. 1, the numbers inside the bracket are the unique number of nodes and the normal numbers are the unique number of elements.

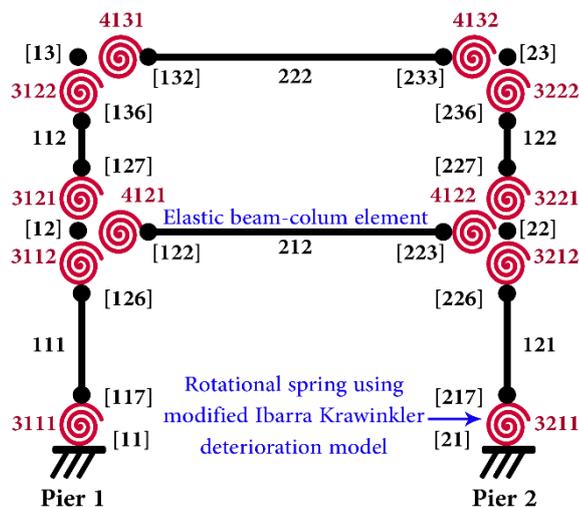


Fig. 1: The second method for modeling, the elastic beam-column elements are connected using rotational springs [51]

The performance of the concentrated plastic hinges in the Ibarra and Krawinkler model is only flexural without considering any axial force. Therefore, considering these hinges for the columns with a large amount of axial force produces some inaccuracies, thus, it is better to use them for the beams. On the other hand, calculating hinge rotations in a distributed plasticity model with nonlinear beam-column elements bears some errors that makes it less applicable in research.

In this research, the frames are controlled for maximum drift ratios. Therefore, if there is no significant difference in the drift response of the two models, by accepting the rotation error of the column joints in the Ibarra and Krawinkler models, this model can be used to investigate the structural response at different levels of seismic hazard. It should be noted that due to the lateral resisting system of this study, which is a moment-resisting frame with a slight axial load in its columns, it is expected that the use of concentrated hinges will cause insignificant error.

2.1. Properties of the frames

The properties and assumptions of the frames with 3, 7 and 12 stories, named as SF3, SF7 and SF12 respectively, used in this research are as follows.

- The initial design of these frames are carried out in ETABS using ASCE07-10.
- All of the frames are two dimensional with three spans and have joint supports.
- In all cases, the dead load is $7 \frac{\text{kN}}{\text{m}^2}$ and the live load is $2 \frac{\text{kN}}{\text{m}^2}$.
- HEA sections and HEB sections are used for beams and columns respectively, in order to design the frames.
- The yield stress of steel is equal to 240 MPa and the young modulus is equal to 200 GPa.
- The soil type is C and the spectral accelerations are 1.5 and 0.6 for periods 0.2 and 1 seconds, respectively. Other required parameters are considered according to ASCE 07-10.
- In all of the frames, the story height is 3.2 meters and the length of spans are 6 meters.
- The length of the spans perpendicular to the frame in all of the frames are 6 meters.

It should be noted that the properties of HEB and HEA sections are continuous in order to be able to optimize the problem and these continuous section properties are obtained by interpolating the section properties as a function of the area of the available sections. It was studied that replacing continuously optimized sections with the closest existing ones does not produce much difference in the results.

The geometry of initially designed SF7 frame and the section properties according to ASCE 07-10 is shown in Fig. 2.

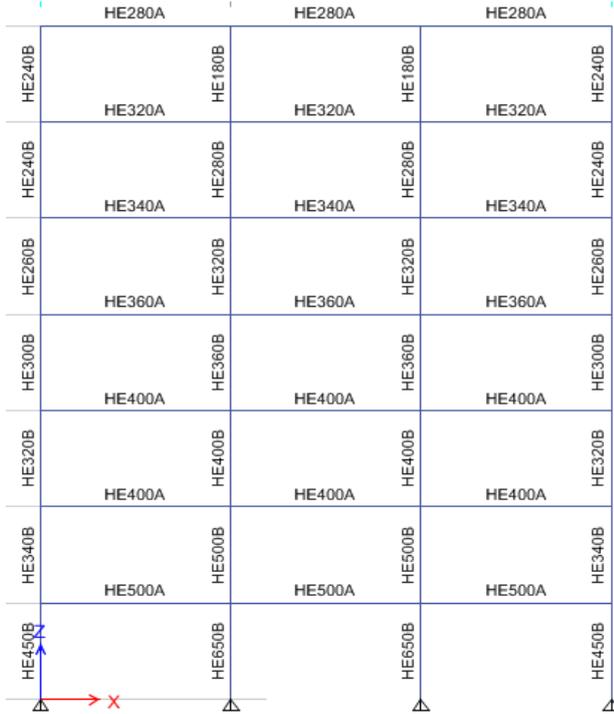


Fig. 2: The SF7 frame

2.2. Performance-based design of steel frames using endurance time method and the uniform deformation theory

This method involves a few steps as follows:

- 1- Determination of performance goals corresponding to a specific seismic hazard level
- 2- Design of the primary structure
- 3- Running structural analysis using endurance time method and evaluating structural performance
- 4- Changing section properties of the frame to make its drift distribution uniform. This is done using the uniform deformation theory [18,53]. This step is executed until the desired performance is achieved.

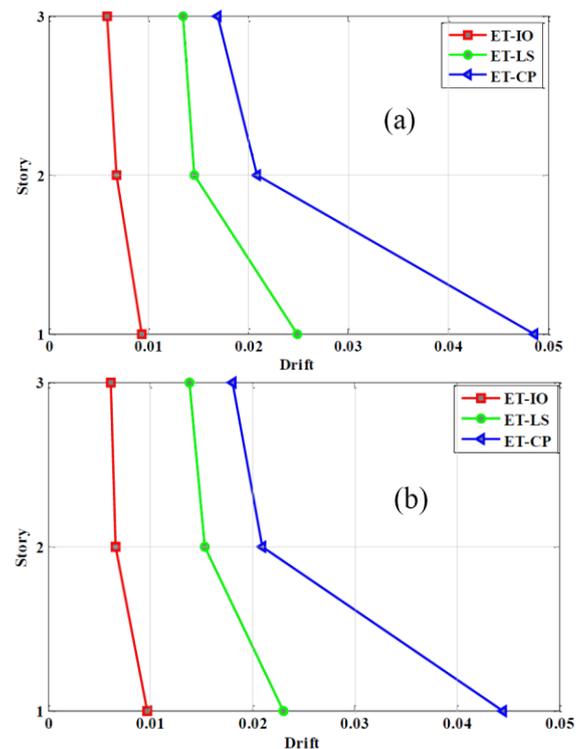
In this research, using the uniform deformation theory proposed by Hajirasouliha and Moghaddam [53], an optimum design that provides suitable and uniform drift for all stories is presented. This algorithm leads to optimal distribution of stiffness and strength. To do so, for the assumed seismic hazard level, the equivalent target time of endurance time method is calculated and the distribution of deformation at different stories is achieved at that time. The distribution is compared with the target value and the section properties of the elements of the structure are modified in such a way to make the deformation more compatible with the target value. This is done by increasing section properties for the stories with larger deformation than the target values and decreasing section properties for the stories with smaller deformation than the target values. The coefficient of variation of the deformation is calculated and if it is smaller than the acceptable limit, the optimization stops.

3. Results and discussion

In this section, the results of the analyses based on the endurance time method and time history analysis together with uniform deformation theory are presented and discussed. ETA40g series of endurance time acceleration functions are used in this study. The spectral acceleration record of ETA40g in 10 second matches the design spectral of ASCE 07 with $S_s = 1.5$, $S_1 = 0.6$, $F_a = 1$, $F_v = 1.3$, $T_L = 8$ second and soil type C which is the basis of the frame designs. Also SAC ground motion records are used to evaluate the accuracy of endurance time method which is compatible with ETA40g series.

3.1. The selection of the model for seismic performance design

In order to compare the accuracy of structural modeling, SF3, SF7 and SF12 frames are modeled with distributed plasticity (using nonlinear beam-column) and concentrated plasticity (Ibarra and Krawinkler model). Subsequently, the drift responses of these two models in different seismic hazard levels are compared using the endurance time method. The results of these two models are shown in Fig. 3. As it is clear, the maximum response differences in these models are 10 to 15 percent. This discrepancy in all frames is mostly observed in high seismic hazard levels. Therefore, Ibarra and Krawinkler model is a suitable model for predicting the maximum drift of stories in low and medium seismic hazard levels. In addition, this model is acceptable with good accuracy in the high seismic level.



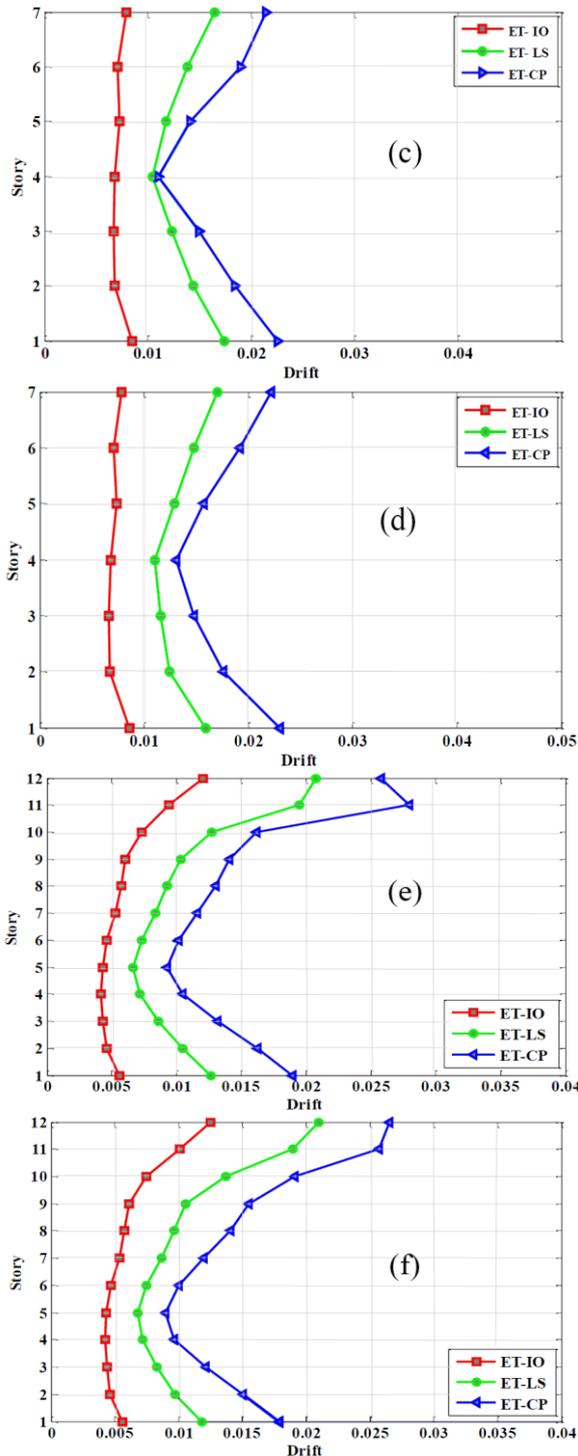


Fig. 3: The comparison of drifts between two models, a) SF3-concentrated plasticity b) SF3-distributed plasticity c) SF7-concentrated plasticity d) SF7-distributed plasticity e) SF12-concentrated plasticity f) SF12-distributed plasticity

3.2. Design of steel frames based on drift control in a specific hazard level

For this purpose, steel moment-resisting frames were designed in accordance with seismic codes. Then, to evaluate the performance of the frames at different levels of seismic hazard, the endurance time method is used and frame response curves are plotted over time. After

comparing the results of the endurance time method and the desired performance goals, necessary decisions have been made to redesign the frame. The response curve in the endurance time method is obtained by calculating the maximum absolute response of the structure at any time. Then, the average response values obtained from these series of acceleration functions are considered as the structural response. The drift response obtained from endurance time analysis for SF3 frame is shown in Fig. 4

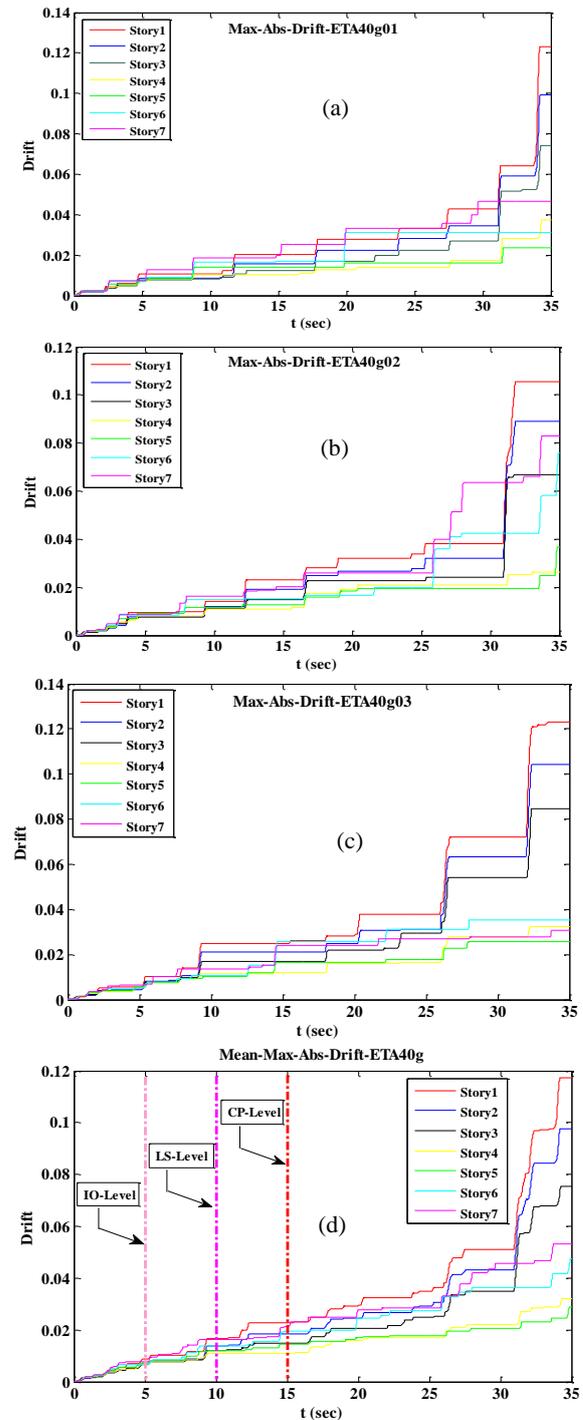


Fig. 4: The response of endurance time method for the drift of the SF7 a) ETA40g01 b) ETA40g02 c) ETA40g03 d) the average ETA40g

As it was discussed before, at 10 seconds of the ETA40g series, the life safety performance level should be achieved. To investigate the collapse prevention performance level, seismic hazard level with the return period of 2475 years should be used, which is 1.5 times the design spectrum. Therefore, the equivalent time for the endurance time method for this seismic hazard level is set at 15 seconds. Another seismic hazard level which is 0.5 times the design spectrum is also assumed in this study. This level is equivalent to the 5 seconds of ETA40g series. The target drift for optimization of the structures at IO, LS and CP performance level is assumed to be 0.7%, 1.5% and 3%, respectively. These values should be obtained at 5, 10 and 15 seconds in endurance time analysis (Fig. 4).

Fig. 5 shows the drift of SF7 frame optimized for IO level at different levels of seismic hazard. As it is shown, the drift at IO performance level is appropriate, and in addition to being below the permissible level (in most stories), the relative displacement distribution is uniform. At the LS level, similar to the IO level, most stories (except the first and the last story), are within the permissible range, except that at this level, the uniformity of relative displacement distribution is no longer observed, indicating an inadequate distribution of materials for this level of performance. At the CP level, all stories are within the permissible range, but similar to the LS level, scattering in response is also observed here. In addition, at the CP level, the response values of the stories are less than the permissible values, indicating an over-design for this level.

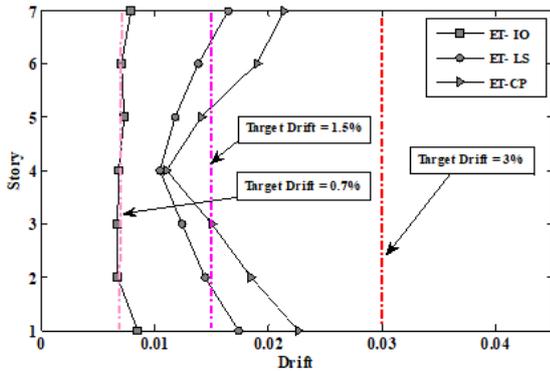


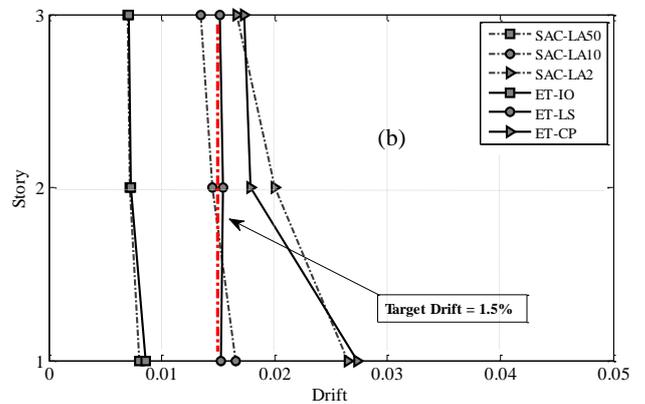
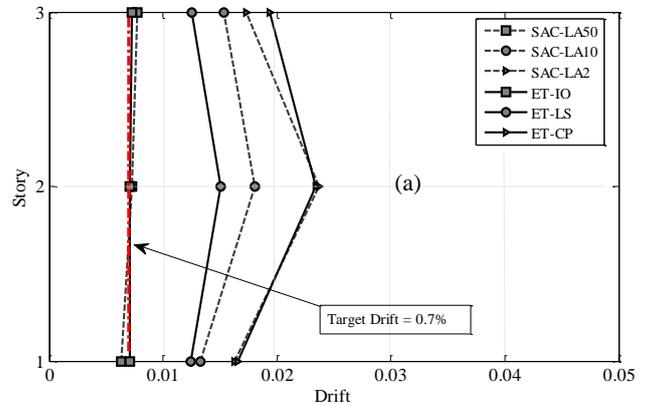
Fig. 5: The drift of the SF7 frame under seismic hazard levels (ET)

Now the question that should be addressed is, how seismic hazard level should be selected for optimization to get better structural response at other levels. For this purpose, SF3, SF7, and SF12 frames were analyzed at different seismic hazard levels by the endurance time method and the corresponding response to each of these levels has been presented in the form of drift. After comparing the response of the structure with permissible values and applying the theory of uniform deformation, it is possible to achieve a structure with a uniform drift distribution at a seismic hazard

level by moving to the optimal sections and redistributing the sections. The theory of uniform deformation in drift control is, to strengthen the story in which its drift is larger than the permitted value and weaken the story in which its drift is less than the permitted value.

Figs. 6 shows the response of SF3 frame at different seismic hazard levels, respectively. In each frame, the objective is to design a uniform relative drift of the structure at a seismic hazard level by accepting a certain amount of error. The error mentioned above is the stopping of the algorithm if the coefficient of variation is less than 0.03. In addition, in order to evaluate the endurance time method, structures with a uniform response designed with the endurance time method were subjected to time history analysis of the Los Angeles SAC project records [52], and the results of both methods were compared. Since comparisons of endurance time analysis results have to be made with the same set of records, the SAC record sets have been scaled so that the average acceleration spectrum of this set of records is consistent with the endurance time spectrum at the desired times.

Figs. 6 illustrates that the endurance time method and time history analysis of the records set at low and moderate seismic hazard levels are well matched, while this adjustment does not exist for high seismic hazard level. Results show that for all frames, if the structure is optimized for moderate seismic hazard, the overall performance at all levels is better than the optimization based on other levels.



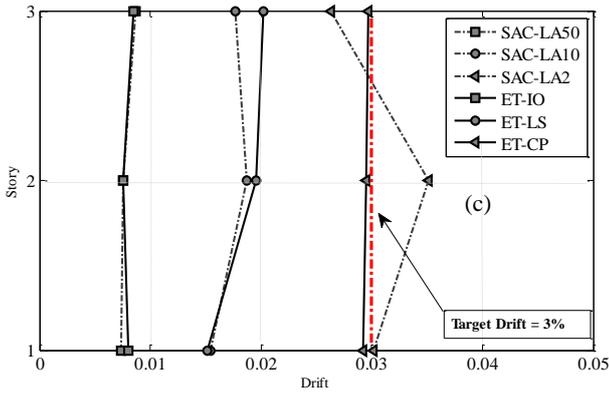


Fig. 6: The drift of the designed SF3 frame by two methods for a) IO b) LS c) CP in different seismic hazard levels

3.3. Optimum design of steel moment-resisting frames based on drift control at different seismic hazard levels simultaneously

As stated before, the optimal design of structures for one seismic hazard level does not ensure the proper performance of the structure at other seismic hazard levels. Therefore, the following algorithm is used to optimize the drift at all levels of hazard.

- 1- The structure is optimally designed for a low hazard level (here 5 seconds in ETA40g series records) by the endurance time method.
- 2- A damping system with bi-linear behavior is added to all stories of the main structures. The performance of these dampers is to assist structures at higher seismic hazard levels and to create a uniform drift distribution. For this purpose, these dampers should not operate at low seismic hazard levels and should be activated at higher seismic hazard levels. Therefore, the GAP element is added to the dampers. This element performs in a manner that before the drift reaches a certain value, the damper has no effect on the behavior of the structure and its effect is when the drift is greater than the specified value. The gap value is equal to 80% of the maximum displacement of the stories at a low seismic level. In addition, the initial stiffness of the damper is equal to 10% of the lateral stiffness of the middle span of each story and the yield strength of the damper is equal to 2% of the initial stiffness of the damper. The lateral stiffness of the middle span is assumed to be equal to Eq. (10) which is used for frames with fixed supports. More accurate values can be obtained by a trial and error process and in general, it requires more precise studies.

$$K = \frac{24EI_c}{h^3} \frac{12\rho + 1}{12\rho + 4} \quad (10)$$

where h is the span height, I_c is the column moment inertia, E is the elastic modulus, ρ is calculated based on the span length (L), story height (h), beam moment of

inertia (I_b) and column moment of inertia according to Eq. (11)

$$\rho = \frac{EI_b/L}{2EI_c/h} \quad (11)$$

- 3- Given the drift values of the new structure, at the moderate hazard level (10 sec time in ETA40g series records), a target drift is assumed for this level. The target drift should be considered with accuracy (about average drift) in order to make optimization process possible. If this value is considered to be high, the dampers cannot weaken the structure and it is impossible to achieve this target drift, and if a low value is selected, the design would not be economical.
- 4- The new structure is analyzed using the endurance time method, and the drift for all stories is calculated in all of the seismic hazard levels.
- 5- The coefficient of variation (CV) of the drift is calculated at the moderate hazard level and the process stops if CV is small (less than 0.05).
- 6- If the coefficient of variation is large, the yield strength and stiffness of the dampers whose relative drift at the moderate hazard level is greater than the target value are modified according to Eq. (12), and the algorithm continues again from step 4.

$$[SD_i]_{m+1} = [SD_i]_m \left[\frac{Drift_i}{Drift_t} \right]^\alpha \quad (12)$$

Where $[SD_i]_m$ is the yield strength of the damper in the story i in the m^{th} step of the algorithm, $Drift_i$ is the drift in the story i , $Drift_t$ is the target drift and α is the convergence power which ranges from zero to one. The best values for convergence power are between 0.05 to 0.15 in this study. After the drift became uniform at a moderate hazard level, the above algorithm is re-used for a high hazard level and the drift of structure at this hazard level also becomes uniform. It should be noted that at this stage, GAP values are selected based on the maximum drift in the moderate hazard level and the coefficient of variation must also be calculated for the high hazard level.

Gap dampers capability for structural control at various seismic hazard levels has been demonstrated in Fig. 7 to Fig. 9. As it is shown, gap damper improves structural performance at moderate to high hazard levels, and with this damper, we are easily able to control structures at higher hazard levels. In addition, it is applicable to structures with different number of stories. It should be noted that the use of this damper increases the axial force in the column, and if the axial column force exceeds the range specified for the displacement control members, the force control requirements for these columns must be checked.

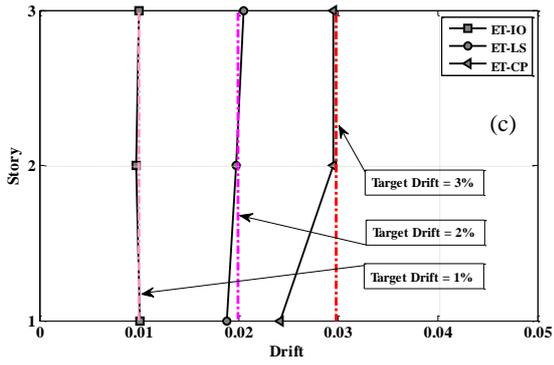
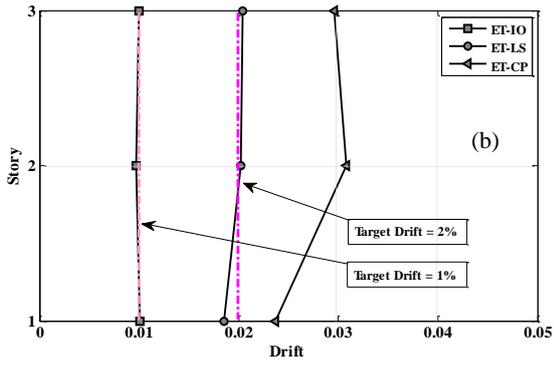
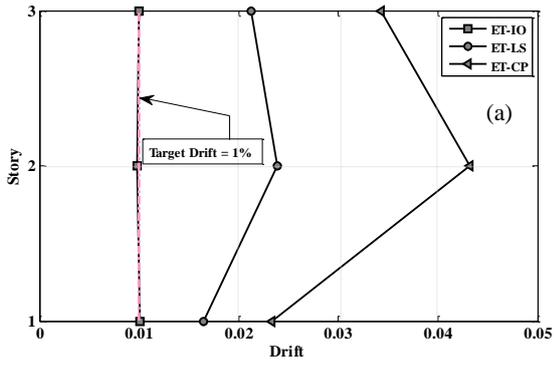


Fig. 7: The story drifts for the SF3 frame rehabilitated with gap dampers in different seismic hazard levels, a) IO b) IO and LS c) IO, LS and CP

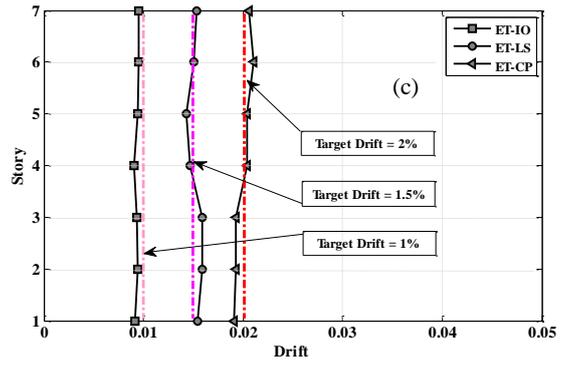
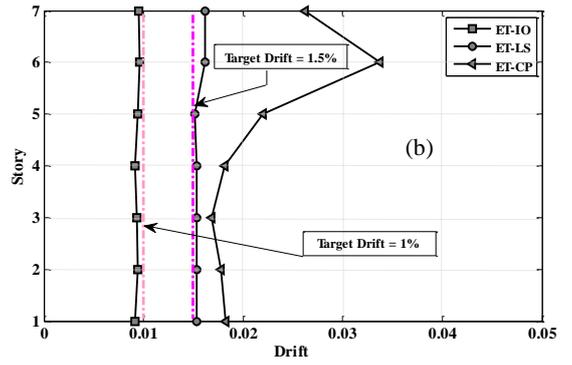
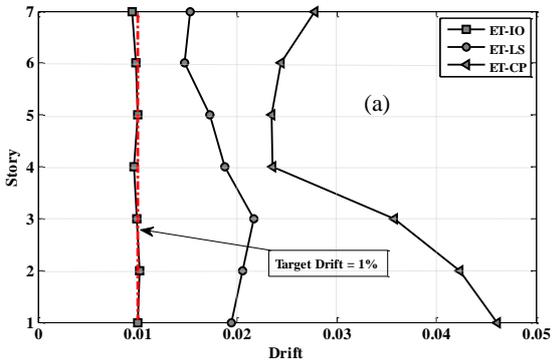
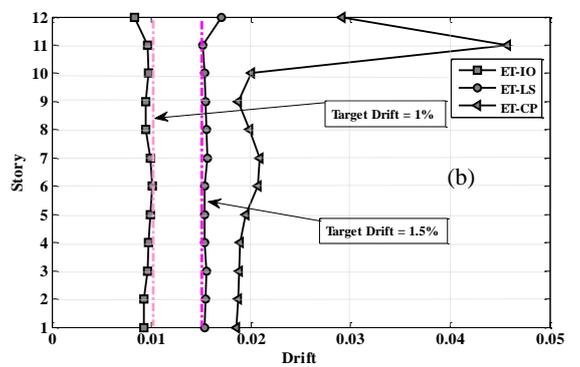
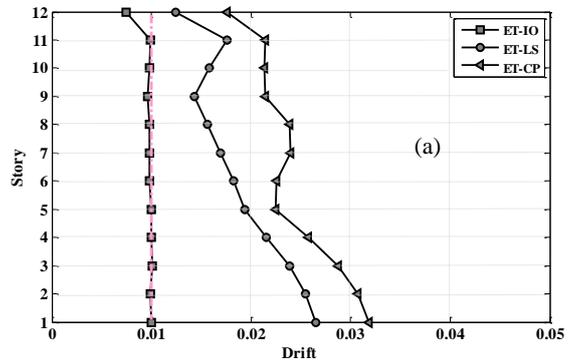


Fig. 8: The story drifts for the SF7 frame rehabilitated with gap dampers in different seismic hazard levels, a) IO b) IO and LS c) IO, LS and CP



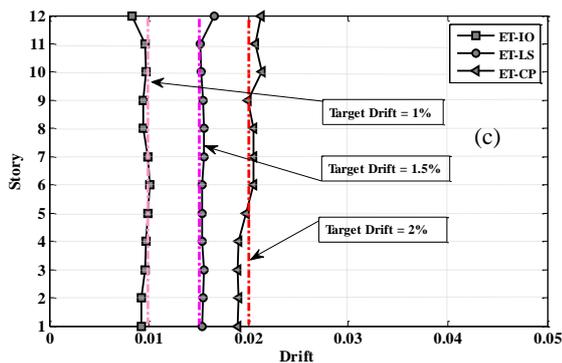


Fig. 9: The story drifts for the SF12 frame rehabilitated with gap dampers in different seismic hazard levels, a) IO b) IO and LS c) IO, LS and CP

4. Conclusion

In this paper, a new and fast method based on endurance time method analysis combined with the uniform deformation theory was proposed to optimize steel moment-resisting frame design. In order to evaluate the effectiveness of this method, three frames with different number of stories were selected, and these frames were optimized at a specific hazard level and different hazard levels using endurance time method and the uniform deformation theory. Results of this study can be summarized as follows:

- Comparison of the results of optimized frames using time history analysis and endurance time method showed that there is a good agreement between the results of these two methods at low and moderate hazard levels; however, this consistency was not quite appropriate for the analyses at high hazard level.
- The uniform deformation theory could be used effectively to optimize steel moment-resisting frames with different stories, but uniform distribution of the drift was obtained in just one seismic hazard level, and it was not possible to have uniform drift distribution at different seismic hazard levels simultaneously. To have the best performance at different hazard levels, it was reasonable to optimize the structures at the moderate seismic hazard level.
- In order to optimize the structure at all seismic hazard levels, the GAP dampers were used. These dampers should be effective after a specified drift at the lower seismic hazard level. Using such dampers, it was theoretically possible to have uniform drift distribution at different seismic hazard levels.

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